STOUGH

A Sewage Treatment Plant for Champaign

Municipal & Sanitary Engineering

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1913

A SEWAGE TREATMENT PLANT FOR CHAMPAIGN

BY

GLENN HOWENSTEIN STOUGH

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

MUNICIPAL AND SANITARY ENGINEERING

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

GLENN HOWENSTEIN STOUGH

ENTITLED A Sewage Treatment Plant for Champaign

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Municipal and Sanitary Engineering.

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INTRODUCTION

Champaign, Illinois has a system of sanitary sewers which served a population of 12,421 in 1910. The sewage is emptied into Salt Fork which is the only available outlet. This stream has a drainage area of 63 square miles above the disposal site. Between Champaign and Danville, where the Salt Fork empties into the Vermilion River, it flows through a populous farming country and several towns are located on its banks.

Figuring on the basis of 1 cubic foot per second per square mile for the ordinary flow and 0.1 cubic foot per second per square mile for the ordinary minimum flow the ordinary flow would be 63 cubic feet per second and the ordinary minimum flow would be 6.3 cubic feet per second. Streams of this size in Illinois, however, are occasionally dry for long periods during the summer months.

Since Urbana also empties its sewage into the same stream about half a mile above the Champaign sewer outlet a total population of approximately 25,000 inhabitants use Salt Fork for sewage disposal. This makes the stream flow only 2.5 cubic feet per second per 1,000 inhabitants at ordinary flow and only 0.25 cubic feet per second per 1,000 inhabitants at times of ordinary minimum flow.

Mr. Hering, Mr. Stearns, and Mr. Goodnough all place the limit of dilution below which nuisance is almost sure to result at 3 cubic feet, or upward, per second per 1,000 inhabitants. Therefore some purification is necessary at times of ordinary flow if nuisance is to be avoided.

Observations were made on April 5th and 6th 1913 on the

quantity and quality of sewage ~~flow~~. These observations were made shortly after a long period of wet weather when the flow was the maximum that it would be desirable to treat since when it becomes larger the flow in Salt Fork will be adequate to take care of it by dilution.

The quantity of flow was computed from observations of depth and velocity in a 600 foot stretch of the outlet sewer which is an 18 inch pipe on a 0.1% grade. Samples were taken by lowering a small pail into the sewer and pouring a half pint portion of this quantity into bottles for later analysis.

The results of these observations are shown in the table below.

Quantity of Flow.

Time	Velocity ft.per sec.	Depth of Flow inches	Discharge gal.per day
2:00 P.M.	2.105	12.0	1,750,000
3:05 P.M.	2.115	12.25	1,800,000
4:15 P.M.	2.085	12.25	1,800,000
5:15 P.M.	2.145	12.125	1,780,000
6:25 P.M.	2.145	12.25	1,800,000
8:30 P.M.	2.225	12.625	1,880,000
9:30 P.M.	2.225	12.25	1,800,000
10:30 P.M.	2.225	12.125	1,780,000
3:30 A.M.	2.077	12.0	1,750,000
4:30 A.M.	2.250	11.0	1,540,000
5:30 A.M.	2.150	11.0	1,540,000
6:30 A.M.	2.210	11.0	1,540,000
7:00 A.M.	2.150	10.75	1,490,000
9:30 A.M.	2.160	11.5	1,640,000

The average flow computed from these observations is 1,700,000 gallons per day.

The results of the analysis made by the Illinois State Water Survey on the samples are as follows;

Time	Chlorine parts per million
1:45 P.M.	76
4:15 "	70
6:30 "	72
8:45 "	49
10:30 "	55
3:30 A.M.	49
6:45 "	43
9:30 "	61

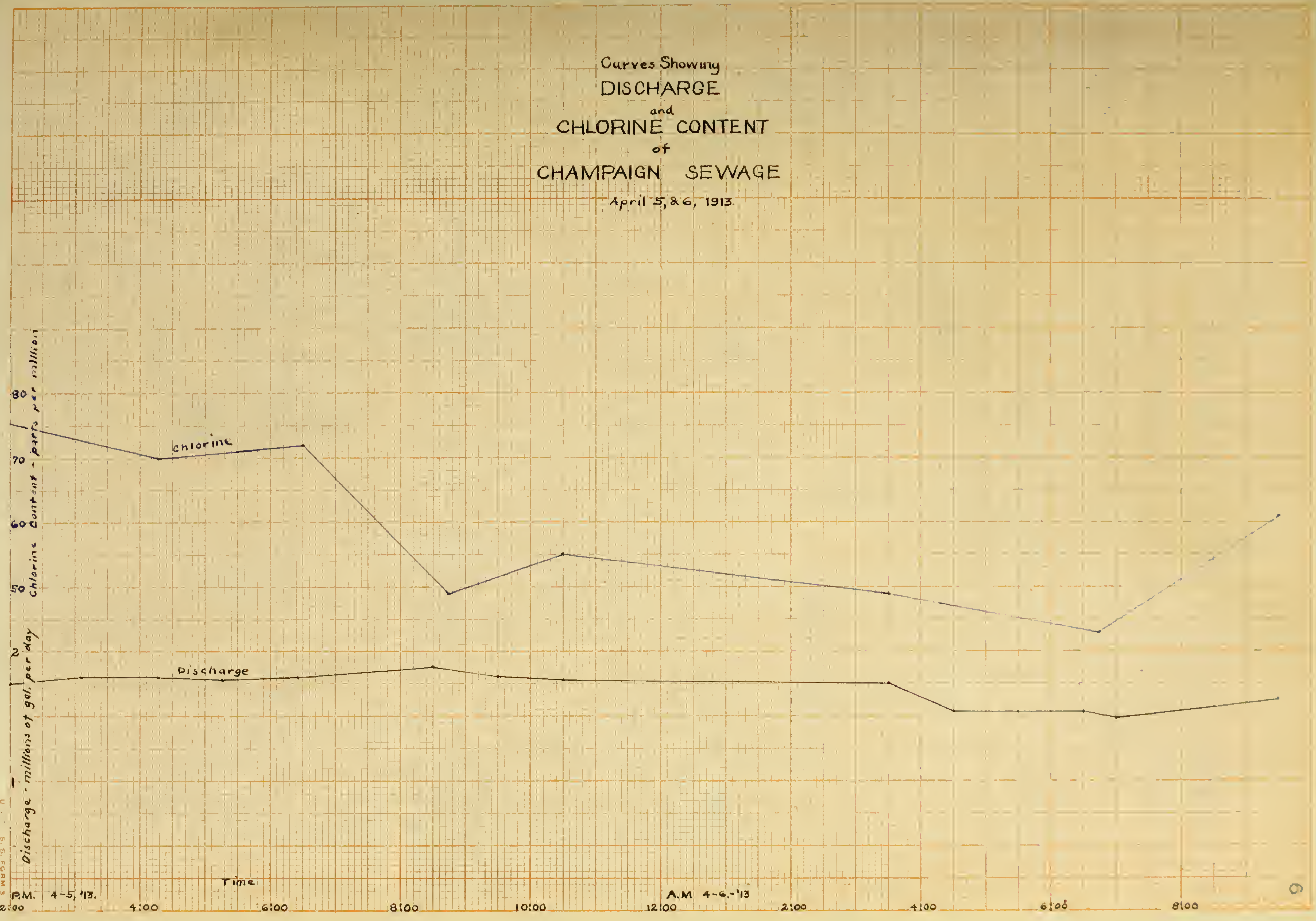
A composite sample using equal parts of the eight samples taken gave the following analysis;

Turbidity	20
Color	30
Odor	3d

	Total	Dissolved
Residue on evaporation	764.	748.
Chlorine in chlorides	61.	61.
Oxygen consumed	20.0	17.2
Nitrogen as free ammonia	12.80	9.00
Nitrogen as albuminoid ammonia	1.40	0.880
Nitrogen as nitrites	1.250	
Alkalinity	302.	
Total organic nitrogen	2.80	2.08

Some of this data is shown graphically on page 6.

The above analysis indicates that the sewage is weak,


Curves Showing
DISCHARGE
and
CHLORINE CONTENT
of
CHAMPAIGN SEWAGE
April 5, & 6, 1913.
Chlorine
Discharge
Chlorine Content - parts per million
80
70
60
50
Discharge - millions of gal. per day
2
1
Time
P.M. 4-5, '13.
A.M 4-6.-'13
2:00
4:00
6:00
8:00
10:00
12:00
2:00
4:00
6:00
8:00
U. S. S. FORM 3

and old by the time it reaches the disposal site.

A study of the construction and condition of the sewer system would lead to the same conclusions. Champaign is built on quite level ground,necessitating very flat grades for the sewers. The outlet sewer is 18 inch vitrified pipe, laid on a 0.1% grade, and is 2.0 miles long. The computed time of flow in this portion is 1 hour and 30 minutes which would be the age of the sewage from the nearest house connection. There is a great deal of leakage in the system as is evidenced by the discrepancy between the observed quantity of flow and the records of the water works pumping station and the wide variation in the flow at times of wet and dry weather.

The present disposal plant is a tank designed by Professor A. N. Talbot in 1895 when the city had a population of 7,000, one half its present population. The tank was constructed in 1897

It has two compartments under one roof, each 37 feet long, 8 feet wide and 5 feet deep below the flow line, giving a combined capacity of of 2,960 cubic feet. At the time when the quantity of flow was determined as 1,700,000 gallons per day, which would give a theoretical time of flow of 20 minutes, a large quantity of fluorescein was introduced into the influent pipe and the time for its complete discharge was observed as less than 4 minutes. The great discrepancy is accounted for by the fact that the tank is so nearly filled by scum and deposits. The scum consists principally of paper. It attains a maximum thickness when it reaches the depth of the scum boards which it does approximately one week after cleaning and from that time on until the next cleaning practically all the solids go straight through.

It is therefore quite evident that a new disposal plant is a necessity and the object of this thesis is to design a disposal plant for Champaign which will give the desired degree of purification.

DESIGN

I. PROCESS OF PURIFICATION

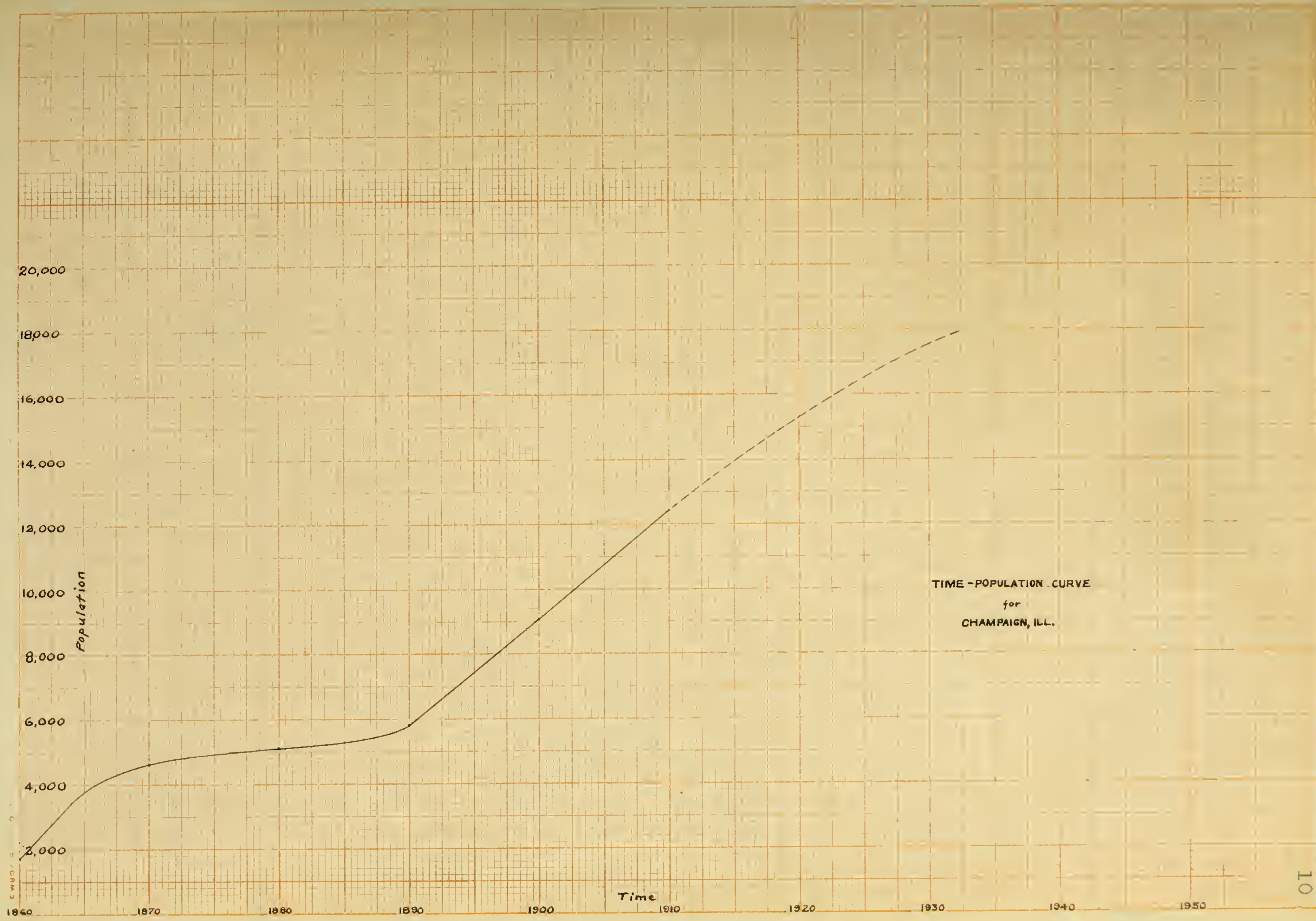
1. Quantity to be Taken Care of.

The census reports for Champaign from 1860 to 1910 have been as follows;

Year	Population
1860	1,727
1870	4,625
1880	5,103
1890	5,839
1900	9,098
1910	12,421

These figures are plotted on page 10 and the curve is projected into the future. Since all parts of the plant to be built are easily subject to enlargement it is thought advisable to build for only 10 years in the future. From a consideration of the curve on page 10 the population in 1923 will probably be 16,000.

If the sewage flow increased in proportion to the population the flow in 1923 would be 2,140,000 gallons per day. If the increase in population added only 60 gallons per capita per day, the water consumption rate for Champaign, to the sewage flow it would be only 1,898,000 gallons per day in 1923. The leakage is largely ground water as is shown by the continued high rate of flow after a rain and the low rate of flow during long dry periods. This leakage is probably mostly into the main sewers which will be extended very little to accomodate an increase in population and since it is to be expected that even an increase


TIME - POPULATION CURVE
for
CHAMPAIGN, ILL.
Population
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
Time
1860
1870
1880
1890
1900
1910
1920
1930
1940
1950

in house connections will somewhat augment the leakage it is not likely that either of the rates given above would obtain but that the flow would be something between them. Therefore it is thought advisable to design the plant for a sewage flow of 2,000,000 gallons per day.

2. Available Fall

The elevation of the bed of the Salt Fork at the disposal site is 50.20 feet. The elevation of the ordinary flow is about 50.70. The elevation of the maximum high water that has occurred in several years is 59.20. When 200 cubic feet per second are flowing, or 8 cubic feet per second per inhabitant using Salt Fork for sewage disposal, the elevation of the surface of the stream is about 51.70 and it is not deemed necessary to have complete purification when the stream rises higher than this. Since the sewage arrives at the disposal site at elevation 59.00, a fall of 7.3 feet is available without pumping as long as perfect operation of the plant is desirable.

3. Selection of Parts.

A. Preparatory Treatment

As the sewage contains no heavy solid mineral matter, no grit chamber or other form of preparatory treatment before admitting the sewage to the tanks is deemed necessary.

B. Filters

Since a high degree of purification is thought necessary some form of secondary treatment is considered necessary.

The disposal site owned by the city at present is inadequate for the installation of sand filters as approximately 25 acres would be necessary. This would be very expensive because of the

high cost of land, approximately $150.00 per acre, and the high cost of sand, approximately $6.00 per cubic yard. Contact beds or trickling filters would be much cheaper.

In comparing the cost of contact beds and trickling filters only the excavation, filtering material, power,and sprinkling system are considered as the other charges such as walls, under-drains,etc., will be very nearly equal for either system.

The data used, and assumptions made for computing the cost of contact beds were as follows;

Contact beds can be operated with the available fall, 7.3 feet, without pumping.

The current rate of interest is 4%.

The cost of suitable stone is $1.65 per cubic yard.

The cost of excavation would be $0.25 per cubic yard.

As the surface of the ground is approximately level with the proposed surface of the beds the last two items may be combined for purposes of computation.

Little data are available on the life and cost of cleaning contact beds. It is estimated that for Champaign the beds would require cleaning every five years at a cost of $0.50 per cubic yard.

It is assumed that the material would have 25% of voids after having been in use for some time and that a contact every 16 hours would be the desirable rate of operation.

The amount of sewage which would have to be treated at each filling of the entire area would therefore be 2/3x2,000,000 gallons, or 1,333,000 gallons. The volume of voids necessary would be 1,333,000÷7.5=178,000 cubic feet. The volume of filter-

ing material and excavation necessary would be 4x178,000 cubic feet, or 26,300 cubic yards.

Cost of material and excavation, 26,300x$1.90= $50,000

Cost of cleaning, $\frac{\$0.50}{5}$ =$0.10 per cubic yard per year.

Cost of cleaning whole area, 26,300x$0.10=$2,630.00 per year.

Capatalized cost of cleaning, $\frac{\$2,630}{0.04}$ = $65,600

Total capatalized cost of contact beds, $115,600

The data used,and assumptions made,for computing the cost of trickling filters were as follows;

The cost of stone,excavation, and the current rate of interest is the same as before.

For trickling filters the sewage would have to be pumped about 9 feet to have the beds 6 feet deep and obtain a maximum head of 8 feet on the nozzles. Power can be obtained for 3.5¢ per kilowatt hour when used at a rate of of from 2,000 to 3,000 kilowatt hours per month.

The rate of flow for pumping will be taken as 1,800,000 gallons per day as this will provide an ample average for some time in the future and present indications are that the cost of electric power will be materially decreased in Illinois in the near future.

The cost of the motor is $25.00 per horse power.

The cost of a cetrifugal pump of 2,000,000 gallons daily capacity is $300.00.

The efficiency of the plant would be 60%.

The cost of maintainence and repairs on the plant would be $10.00 per year.

The cost of the nozzles and distributing system is \$25,000 per acre.

From experience to the present time with trickling filters, they may be expected to be permanent if a fairly clear sewage is treated.

The rate of filtration is 2,000,000 gallons per acre per day.

Area required, 1 acre, or 43,560 square feet.

Volume of filtering material and excavation required, 6x43,560= 261,360 cubic feet= 9,680 cubic yards.

Cost of filtering material and excavation, 9,680x\$1.90=\$18,400

Cost of distributing system and nozzles, \$25,000

Pumping would require, $\frac{1,800,000 \times 62.5 \times 9}{7.5} = 135,000,000$ foot pounds per day.

Power required, $\frac{135,000,000 \times 100}{60} = 225,000,000$ foot pounds per day = 4.73 horse power = 3.53 kilowatts.

2,540 kilowatts per month would be required, therefore the 3.5¢ rate would obtain.

Cost of power, 12x2,540x\$0.035=\$1,065 per year.

Capitalized cost of power, $\frac{\$1,065}{0.04} =$ \$26,700

Cost of motor, 4.73x\$25=\$118.

Capitalized cost of motor and pump,

$$(\$118+\$300)+\frac{\$10}{0.04}+\frac{\$118+\$300}{(1+0.04)^{10}-1} = \$1,500$$

Total capitalized cost of trickling filters, \$71,600

The above computations show that the trickling filters would cost less than half as much in terms of the capitalized cost and even if attendance at \$60.00 per month were required

for the trickling filters and none were required for the contact beds the difference would still be $48,500.

In addition to their lower cost the degree of purification is much higher with trickling filters than with the contact beds Therefore the former will be selected.

C. Tanks

Imhoff tanks were selected for the preliminary treatment due to the greater ease of sludge disposal, higher degree of purification, and slight possibility of sludge being carried over on to the filters, which would produce clogging.

4. Location of Parts and Routing of Sewage.

The location plan is shown on page 30 and the general plan is shown on page 31.

The tanks will be located 20 feet west of the old tank. This is the lowest ground on the site and will minimize the amount of excavation necessary. A manhole will be built on the present sewer line 140 feet south-westerly of the present manhole at the old tank and a line of 18 inch vitrified tile will be laid north from it to the south-west corner of the Imhoff tanks. This manhole will be provided with gates so that all the flow will ordinarily go through the Imhoff tanks but may be diverted through the old sewer directly into the old tank.

The sludge will be disposed of by dilution at times of high water since flows in Salt Fork of 200 cubic feet per second, which will be ample to take care of the sludge, can frequently be expected. A sludge pipe, connected to each of the sludge compartments, through which sludge will flow by gavity will run directly north from the tanks into the middle of Salt Fork.

The sewage will leave the Imhoff tanks at the north-east corner through a 20 inch vitrified tile pipe which will be laid directly to the north end of the old tank where a new inlet, similar to the present inlet at the south end, will be cut for it. The old tank will be used as a reservoir to collect the sewage from the Imhoff tanks between times of pumping. The pumps will be made to start and stop automatically, their action to be controlled by the sewage level in the reservoir. If the pump is shut off or if it fails to operate the sewage will flow through the present outfall pipe of the old tank directly to Salt Fork, thus by-passing the dosing chamber and filters.

From the reservoir the sewage will be pumped to a dosing chamber at a higher level through a 10 inch cast iron pipe. The dosing chamber will be located 25 feet north of the old tank. It is to be designed to vary the head on the nozzles from 2 to 10 feet. Its action will be controlled by an automatic siphon which will be connected to a 20 inch cast iron pipe. This pipe will conduct the sewage to the filters.

The west line of the filters will be 33 feet east of the old tank. The north line will be about 50 feet south of the south bank of Salt Fork. The filters will be divided into 4 beds and the main drain will run north between the east pair and the west pair. The main drain will be of 18 inch vitrified tile on a 0.4% grade, through which the treated sewage will flow by gravity into Salt Fork. The elevation of the flow line at the point of discharge will be 51.70.

II. TANKS.

1. Rate of Flow and Sludge Storage Capacity.

The tanks are designed for a storage period of 4.1 hours and a sludge storage capacity of 1.83 cubic feet per capita. The time of flow is a little higher than the practice has been with Imhoff tanks but the tendency lately has been toward longer periods of storage. It is thought that 1.83 cubic feet per capita for sludge storage will give a sludge storage capacity of 6 months which gives the maximum sludge digestion that is economical.

The following computations will show that a storage period of 4.1 hours and a sludge storage capacity of 1.83 cubic feet per capita were obtained;

$$\text{Period of storage} = \frac{\text{capacity of tanks}}{\text{quantity of sewage treated}}$$

$$= \frac{\text{cross-section} \times \text{length}}{(2,000,000) \div (7.5 \times 24)} = 4.1 \text{ hours.}$$

$$\text{Sludge storage per capita} = \frac{\text{volume}}{\text{population}} = \frac{29,160}{16,000} = 1.83 \text{ cubic feet}$$

The rate of flow will be, $\frac{96}{4.1 \times 60} = 0.386$ feet per minute.

2. Method of Operation

The tanks are shown in the drawings on pages, 32, 33, and 34.

There will be three tanks, each 96 feet long and 20 feet wide, and each will have three sludge storage compartments 25 feet in diameter with a maximum depth of sewage of 25 feet.

In order to obtain an even accumulation of sludge in the 3 compartments it will be necessary to reverse the direction of flow through the tanks. For this purpose a trough 2 feet wide and with a depth of flow of 1 foot completly encircles the group of tanks. This trough is provided with stop boards as shown in the figure, by means of which the reversal of flow can be accomplished. The sewage enters (or leaves) the tanks over the weirs

between the tanks and troughs.

Gas vents 6 feet by 2 feet are provided over the centre of each sludge compartment. The stays in the upper portion of the tanks together with the ridges between sludge compartments will serve as baffles. A 6 inch sludge pipe will run from the centre of the bottom of each sludge compartment and out through the wall of the compartment 8 feet below the level of the sewage in the tanks. These pipes are provided with a valve and valve box just outside the compartment and are all connected to the main sludge pipe.

3. Design of the Concrete Constuction

The concrete construction throughout the plant is designed in accordance with the tables and diagrams prepared by Messrs. Turneaure and Maurer and published in their "Principles of Reinforced Concrete Construction." The notation employed is also the same as that used in the above text, namely;-

M= the bending moment to be resisted.

f_s= allowable unit stress in the steel in pounds per square inch.

f_c= allowable unit stress in the concrete in pounds per square inch.

p= the ratio of the area of the steel to the area of the concrete.

d= the distance from the compressive face to the plane of the steel.

b= the breadth of the rectangular beam under consideration.

k= the ratio of the depth of the neutral axis of a section below the top to d.

j= the ratio of the arm of the resisting couple to d.

A= the area of the cross-section of the steel.

R= the smaller of the two quantities; $f_s pj$, and $\frac{1}{2} f_c kj$.

The reinforcement throughout the plant will be of 5/8 inch square steel bars. The centre of the steel is to be placed 2 inches from the tension surface of the concrete in all cases. The concrete will be designed for a unit compressive stress of 600 pounds per square inch and a bond stress 80 pounds per square inch between concrete and steel. The steel will be designed for a unit stress of 16,000 pounds per square inch. The ratio of the moduli of elasticity of the steel and concrete will be taken as 15.

A. Sludge Compartments.

(a) Walls

In the design of the walls of the sludge compartments the following assumptions were made:

1. That the earth pressure is equivalent to that of a liquid weighing 25 pounds per cubic foot;

2. That the variation in the earth pressure may reach a maximum of 4 to 1, or that the moment to design the walls for will be, 3/64 x pressure x diameter^2;

3. That enough reinforcement shall be supplied to take the total tension load due to the internal water pressure;

4. That the thickness of the walls at the top shall be 8 inches and that they shall vary uniformly from the top to the bottom.

The moment due to the earth pressure at the base of the vertical wall on a section 1 foot high would be 3/64 x pressure x diameter^2

$=3/64 \times (18 \times 25) \times 25^2 \times 12$ pounds inches

$=161,000$ pounds inches

$=Rbd^2=95 \times 12 \times d^2$

$d^2 = \frac{161,000}{95x12} = 141$ d=11.9 inches

An effective thickness of 13 inches and an actual thickness of 15 inches will be used.

The same spacing of steel will be continued for six feet above the height for which it is computed. The area of steel required for each 6 foot section was found as follows;

At the base of the vertical wall,-

$M=161,000=Rbd^2=Rx12x13^2$

R=79.5 Therefore, p=0.58%

A=bdp=12x13x0.0058=0.905 square inches for each foot in height, which will be the steel area necessary to resist the moment, also,

$A=\frac{\text{pressure x diameter}}{2 \text{ x } f_s} = \frac{18x62.5x25}{32,000} = 0.88$ square inches per foot in height, which will be the steel necessary to resist the water pressure with no earth backing;

At 6 feet above the base of the vertical wall,-

$M= \frac{3}{64} x12x25x25^2x12=107,000$

$=Rbd^2=Rx12x10.67^2$

R=66 Therefore, p=0.47%

A=12x10.67x0.0047=0.603 square inches, also,

$A= \frac{12x62.5x25}{32,000}=0.585$ square inches;

At 12 feet above the base of the vertical wall,-

$M=\frac{3}{64}x6x25x25^2x12=52,800$

$=Rbd^2=Rx12x8.33^2$

R=64 Therefore, p=0.45%

A=12x8.33x0.0045=0.45 square inches, also,

$A=\frac{6x62.5x25}{32,000}= 0.293$ square inches.

From the above computations it can be seen that the moment due to the earth pressure will govern the area of the steel in

all sections, and the spacing of the bars necessary to supply this area in each section will be as follows;

From the base of the vertical wall to 6 feet above the base, 5 inches centre to centre;

From 6 feet above to 12 feet above, 6 inches centre to centre;

From 12 feet above to 18 feet above, or to the top, 8 inches centre to centre.

Assuming the weight of concrete as 150 pounds per cubic foot, the weight of a section of the wall 1 foot in width would be 2,770 pounds. This will give a unit compressive stress of 15.4 pounds per square inch in the concrete at the base of the wall and if the total load were taken by the soil beneath the wall it would be 2,210 pounds per square foot. This will be reduced since the floor and walls are to be built monolithic and some of the load will be transfered to the floor.

(b) Floor

The floor will be designed capable of bridging a span of 4 feet and will be figured as a simple beam.

$$M=\frac{wl^2}{8}=\frac{62.5x25\ x4^2x12}{8}=37,600$$

$$=Rbd^2=95x12xd^2$$

$d=5.75$ inches.

An effective depth of 6 inches and an actual depth of 8 inches will be used.

The area of steel required is found as follows;

$R=\frac{37,600}{12x36}=87$ Therefore $p=0.63\%$

$A=6x12x0.0063=0.46$ square inches per foot.

The spacing of bars required to supply this area will be 8 inches centre to centre.

The walls are tied to the floor by rods, as shown in the draw-

ing, spaced 12 inches centre to centre.

B. Upper Portion.

(a) Walls

The side walls of the upper portion of the tank between the sludge compartments are figured as simple beams subject to internal water pressure. The span will be 17 feet and the maximum water pressure 219 pounds per square foot.

$M=\frac{219x17^2x12}{8}=100,000$ pounds inches

$=Rbd^2=95x12x\ d^2$

$d=9.4$ inches

An effective depth of 10 inches and an actual depth of 12 inches is used.

$A=12x10x0.0068=0.81$ square inches per foot.

The spacing of the rods to supply this area is $5\frac{1}{2}$ inches centre to centre.

The end walls of the upper portion are figured as simple beams and will be made 12 inches thick to conform to the side walls. The span is 20 feet.

$M=131,300$ pounds inches

$=Rbd^2=R\ x12x10^2$

$R=109.5$ Therefore, $p=1.1\%$

$A=12x\ 10x0.011=1.32$ square inches per foot.

The spacing of rods required is $3\frac{1}{2}$ inches centre to centre.

(b) Floor

The floor of the upper portion of the tanks between sludge compartments will consist of triangular slabs, supported on all sides, but due to the flexibilty of the supports it was decided to make them stronger than they would figure and therefore a depth of 12 inches and spacing of rods of 6 inches in both direct-

ions was arbitrarily selected.

6 inches was selected as the depth of the floor of the upper portion of the tanks inside the sludge compartments and for the walls of the gas vent. Figured as a beam these would support their own weight without reinforcement but to take care of temperature stresses rods spaced 18 inches centre to centre are to be supplied and three rods are to be placed in the bottom of these walls as shown in the drawing.

The concrete stay and baffle wall is to be made four inches thick and to be supplied with reinforcing rods down the centre spaced $3\frac{1}{2}$ inches centre to centre.

Troughs.

The troughs will be supported on concrete brackets with the reinforcement anchored in the tank. The regular spacing for these brackets will be 10 feet centre to centre. The walls and floor of the trough will be made 6 inches thick and the walls will be figured to carry the entire load. The load will be due to the weight of the concrete and water. The load per foot of trough will be,

$0.5x5.5x150+2x1x62.5=538$ pounds.

$M=\frac{538x10^2x12}{2x8}=40,400$ pounds inches

$=Rbd^2=R\ x6x24^2$

$R=11.7$ Therefore, $p=0.1\%$

$A=2.5\ x12x6x0.001=0.180$ square inches.

This would call for less than one 5/8 inch bar but for temperature stresses 3 will be supplied, spaced as shown in the drawing.

III. RESERVOIR,(Old Tank), AND PUMPING PLANT.

The sewage will enter the reservoir through the south end, as at present, when the tanks are being by-passed and,when the tanks are in operation , through a similar inlet at elevation 54.00,

which will be below the weir trough, to be built at the north end.

If the pumps are not in operation the sewage, when it reaches elevation 58.88, will flow over the weir and out through the old outlet, thus by-passing the dosing chamber and filters.

A 10 inch centrifugal pump and a 3.5 kilowatt motor with automatic starting and stopping device will be installed. This device will be set to start the pump when the sewage reaches an elevation 58.5 and to stop it when the sewage reaches an elevation of 56.5. The pump and motor will be set on a vertical shaft with the pump below the high water level to eliminate the necessity of priming.

The suction pipe of the pump will run down the centre of the vertical division wall between the compartments of the reservoir and at its lower end a horizontal hole 12 inches in diameter is to be cut through the wall and shear valves are to be placed at either end so that either of the compartments may be used separately or both together. This will provide for cleaning.

IV. DOSING CHAMBER.

It was decided to design the dosing chamber with a capacity of about 12,000 gallons. This would give alterate periods of operation and rest for the filters of about 5 minutes each. As the depth of sewage in the chamber at high water is to be 8 feet, in order to produce a variation in head on the nozzles of from 2 to 10 feet, the size of the tank selected was 15 feet square. This gives a capacity of 1,800 cubic feet or of 13,500 gallons. Since the batter of the walls is to be placed on the inside in order to reduce the amount of water discharged at the lower heads the total capacity will be somewhat reduced.

The walls were figured as beams with horizontal loading due to water pressure. The load at the base of the wall would be,

8x62.5 pounds per foot.

$M = \frac{8x62.5x15^2x12}{10} = 135,000$ pounds inches

$= Rbd^2 = 95x\ 12x\ d^2$

d=10.9 inches

An effective depth of 11 inches and an actual depth of 13 inches will be used.

A=11x12x0.0068=0.90 square inches

The required spacing for the rods is 5 inches centre to centre. The wall will be made 6 inches thick at the top, 10 feet above the floor. At 5 feet above the floor the spacing of the rods is found as follows;

$M = \frac{3x62.5x15^2x12}{10} = 50,700$ pounds inches

$= Rbd^2 = R\ x12x7.5^2$

R=75.3 Therefore, p=0.55%

A=12x7.5x0.0055=0.5 square inches

The required spacing for the rods is 9 inches centre to centre.

The floor is to be made 8 inches thick in 9 square blocks. There is to be a space 1 inch wide and 4 inches deep between the blocks and between the blocks and walls to be filled with a soft asphalt.

The roof is to be made capable of supporting a live load of 100 pounds per square foot and will be supported at the centre by a beam.

The effective depth of slab required for the roof is 5 inches, actual depth 7 inches. The spacing of the rods required is 12 inches centre to centre.

The ridge beam will have to be made capable of supporting 150x7.5 pounds per foot.

$M = \frac{150x7.5x15^2x12}{8} = 380,000$ pounds feet

$=Rbd^2=95x\ bd^2$

If b is made 12.3 inches the effective depth would be 18 inches and the actual depth 20 inches.

$A=12.3x18x0.0068=1.51$ square inches

4 bars will be required to supply this area.

The foundations are to be carried down to elevation 58.00 in order to have them rest upon the original surface of the ground.

V. DISTRIBUTING SYSTEM.

A 20 inch Miller siphon is to be installed in the dosing chamber which will discharge when the surface of the sewage reaches elevation 70.50 and will draw down to elevation 62.50. This is the size recommended by the makers for the daily capacity of 2,000,000 gallons being designed for.

This will discharge into a 20 inch cast iron pipe which will carry the sewage to the filters. At the west line of the filter beds a 10 inch pipe will lead from it to each of the two west divisions of the filters. The 20 inch pipe will be carried through the west divisions, supported by brackets on the division wall. At the west line of the east divisions it will empty into two 10 inch pipes, one for each of the east divisions of the filters.

The maximum loss of head in the 20 inch pipe will be 0.335 feet under maximum head.

Each of the 10 inch pipes will run along the outside of the wall to the east and west centre line of its division and from that point it will run east and west through that division. The maximum loss of head in the 10 inch pipes will be 1.005 feet.

Every 10 feet laterals will be run from the 10 inch pipes. Nozzles will be spaced 11 feet 1½ inches along these laterals. This spacing will place the nozzles in such a manner that hexagonal

sprays from each nozzle will completely cover the area. Between the last two nozzles the laterals will be 4 inch pipe. Between the next two they will be 5 inch pipe and the remainder will be 6 inch pipe. This will give a maximum loss of head in the laterals of 0.120 feet.

The total maximum lost head in the distributing system will be 1.46 feet. This will provide a fairly even distribution of pressure on the nozzles since the maximum variation in pressure will be only 14.6% of the total.

The nozzles are to be Taylor Hexagonal Spray Nozzles. These are designed to throw a spray approximately 15 feet under the high head but are spaced only 11 feet 1½ inches centre to centre in order to secure sufficient overlapping to give an even distribution of sewage over the bed.

VI. FILTERS.

The filters are to be designed to operate at a rate of about 2,000,000 gallons per acre per day with any variation toward a lower rate. The area required is one acre and for purposes of repairs and for accomodating the under-drains to a small fall it was decided to make four divisions with the main drain running between two sets.

The most economical shape for two beds built with a common division wall is to have the width equal to three-fourths the length. The dimensions, in even feet, which most nearly satisfy these conditions are 100 feet by 130 feet.

The average depth of the filtering material is to be 6 feet. It is to be of broken stone, the sizes of the pieces to be from 2½ to 1½ inches.

The main drain for each bed is to be 12 inches wide and 12

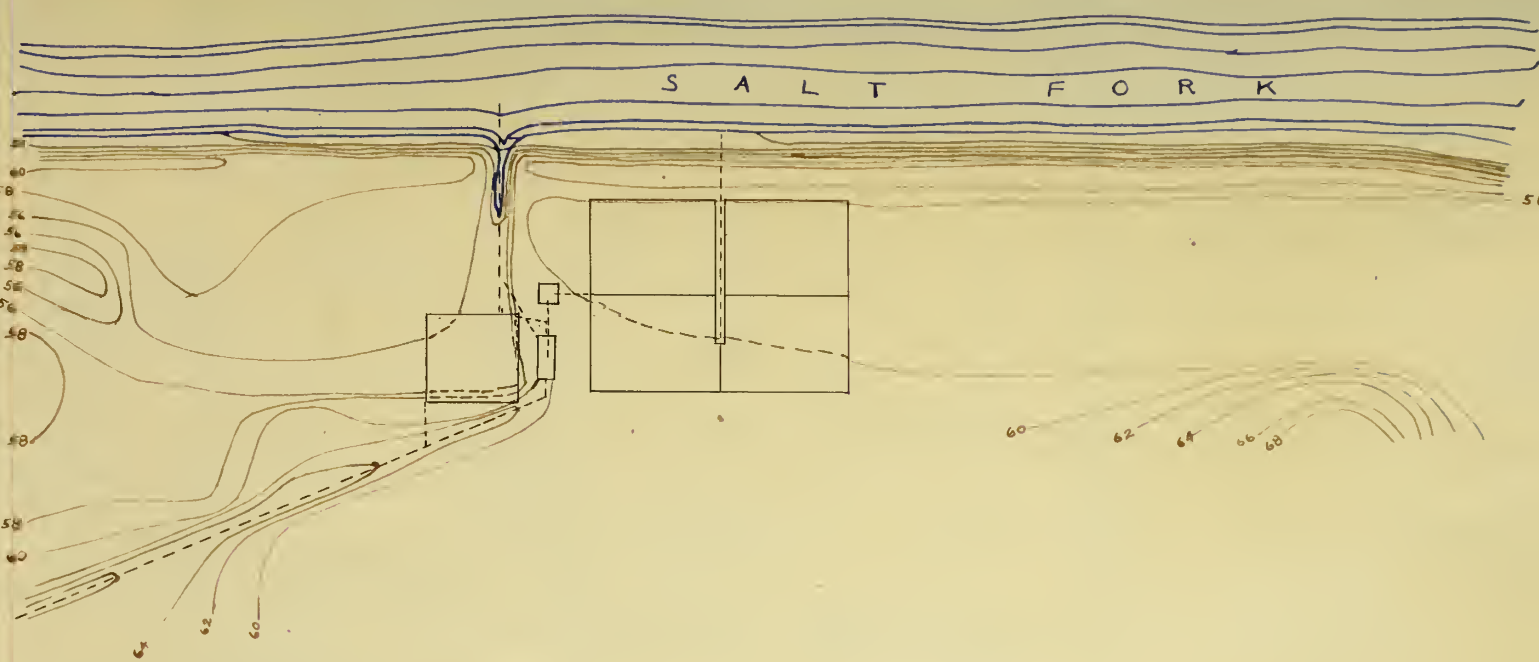
inches deep beneath the floor, and to be laid on a 0.4% grade. These are to run along the east and west centre line of each bed. From these, sub-drains 6 inches wide and 6 inches deep beneath the floor, spaced 14 feet centre to centre will run to the edges of the bed. These are, also, to be laid on a 0.4% grade. The floor is to be of concrete 6 inches thick and will be on a 0.4% grade with a ridge between each two of the sub-drains. The floor is to be covered with 6 inch half tile on top of which the filtering material will be placed. Concrete slabs 6 inches thick and 2 feet long and 12 inches wide for the sub-drains and 18 inches wide for the main drain will be placed across the drain with their edges resting on top of the tile.

Spaced 25 feet apart on the sub-drains will be placed 6 inch air vents, the bottom of the pipe passing through specially made slabs, and the top of the vent to be supplied with orienting funnels. These are intended to supply the beds with good aeration. The main drains will empty into 15 inch tile which will in turn empty into an 18 inch tile on a 0.4% grade that runs directly into Salt Fork.

The walls are to be 6 inches at the top and 18 inches at the bottom. This will not make them stable as retaining walls but, since little variation in pressure is expected between the inside and outside this is not necessary.

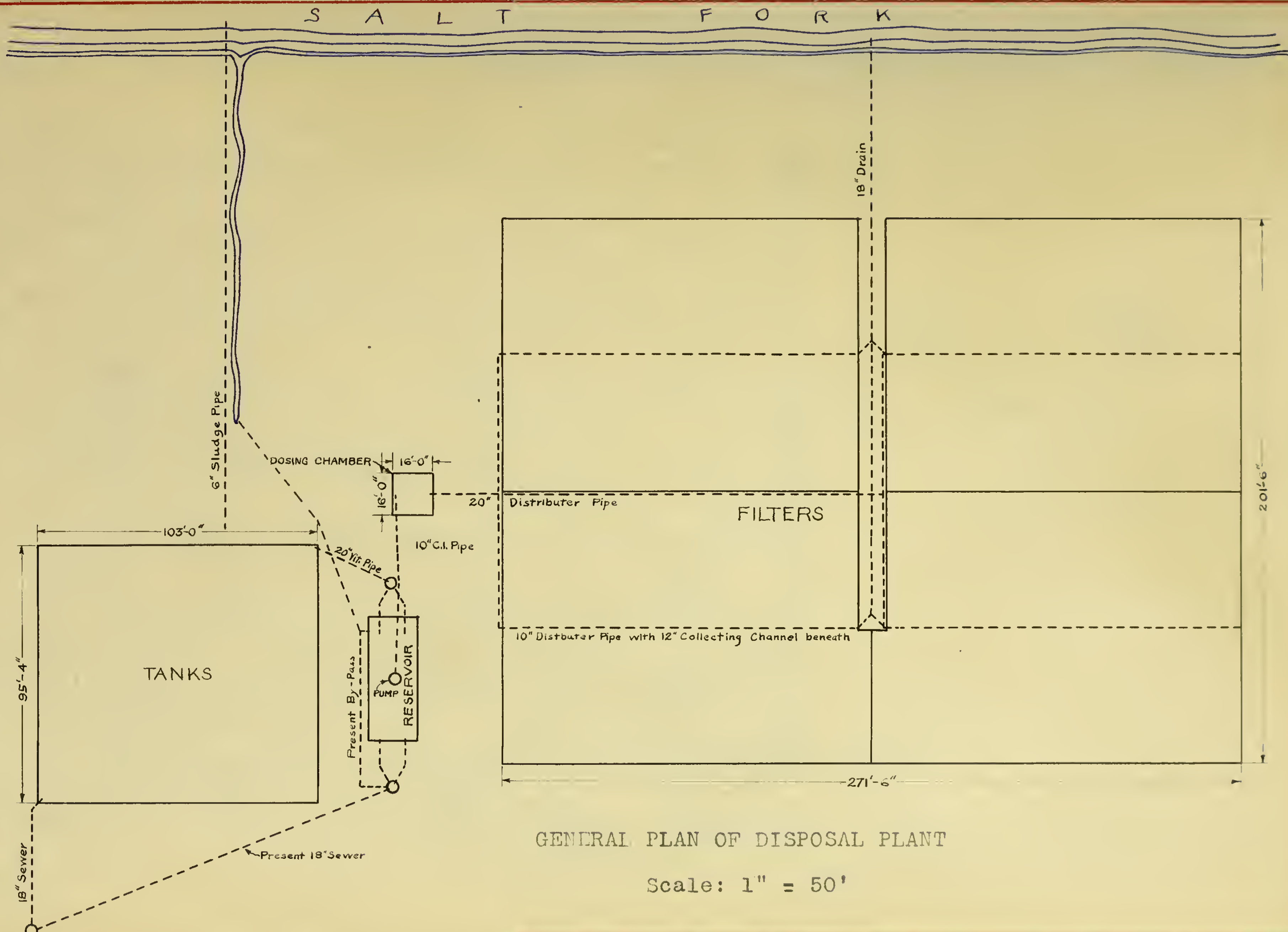
The centre line of the pipes of the distributing system are to be placed 12 inches below the surface of the filtering material and they will be supported from the floor of the filters by rods spaced 11 feet along the axis of the pipe. $\frac{3}{4}$ inch rods will be used for the laterals and $1\frac{1}{2}$ inch rods will be used for the 10 inch pipe.

It is thought that the above design with additions to the units, as necessary, will be such as would take care of the sewage disposal problems of Champaign for many years in the future.



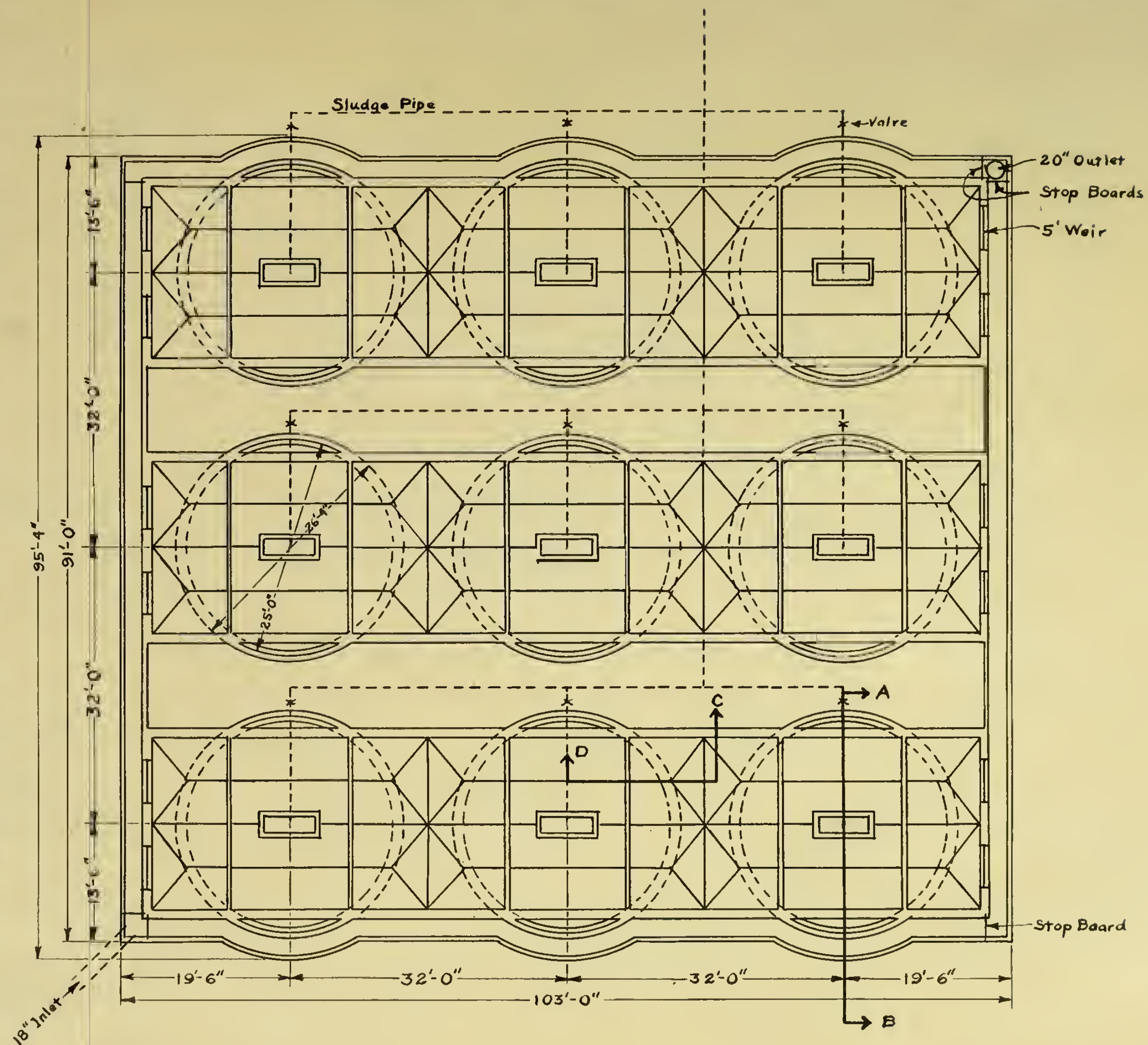
GENERAL LOCATION PLAN
for
CHAMPAIGN SEWAGE DISPOSAL PLANT

Scale: 1"=200'



GENERAL PLAN OF DISPOSAL PLANT

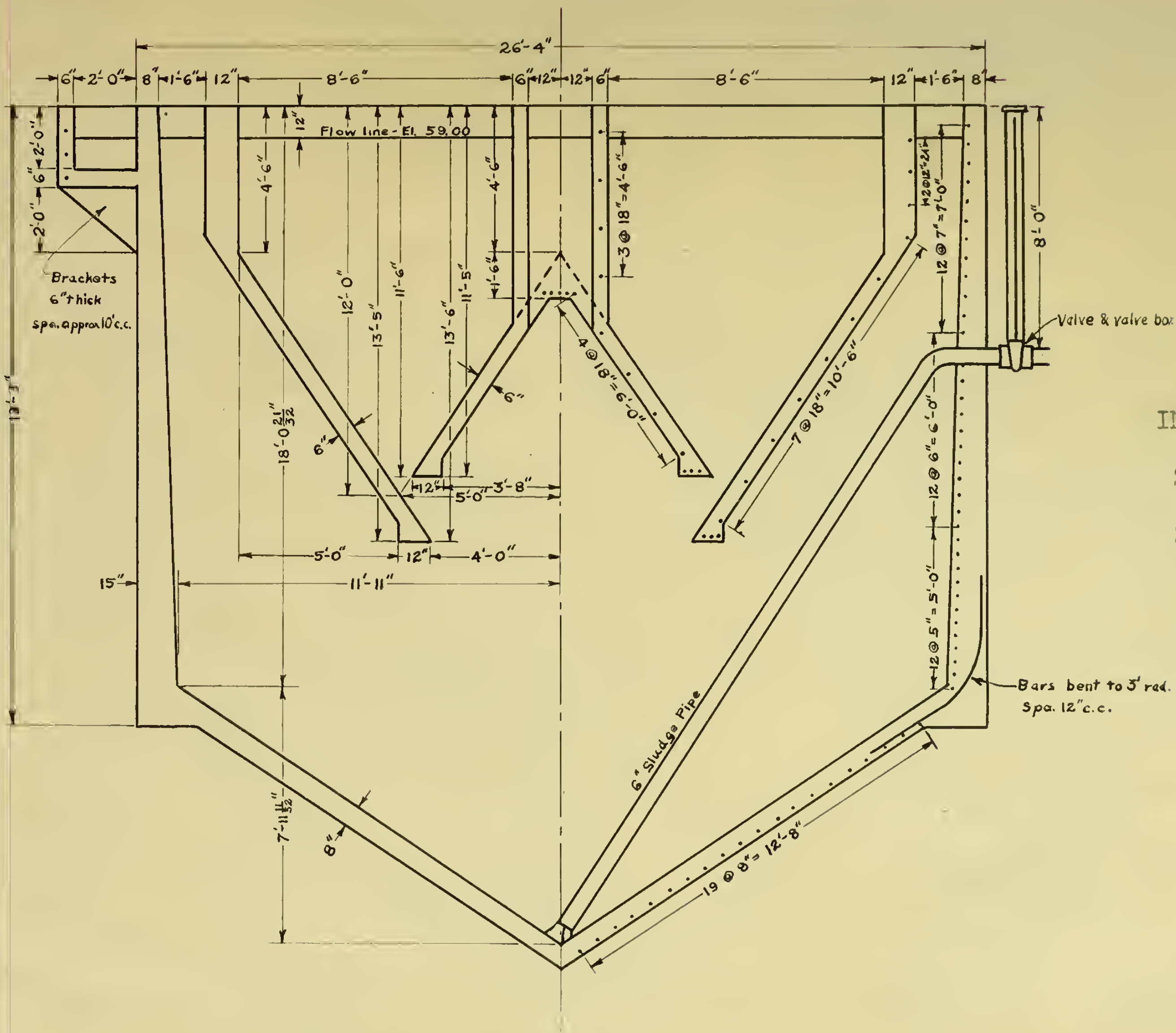
Scale: 1" = 50'



IMHOFF TANKS

PLAN

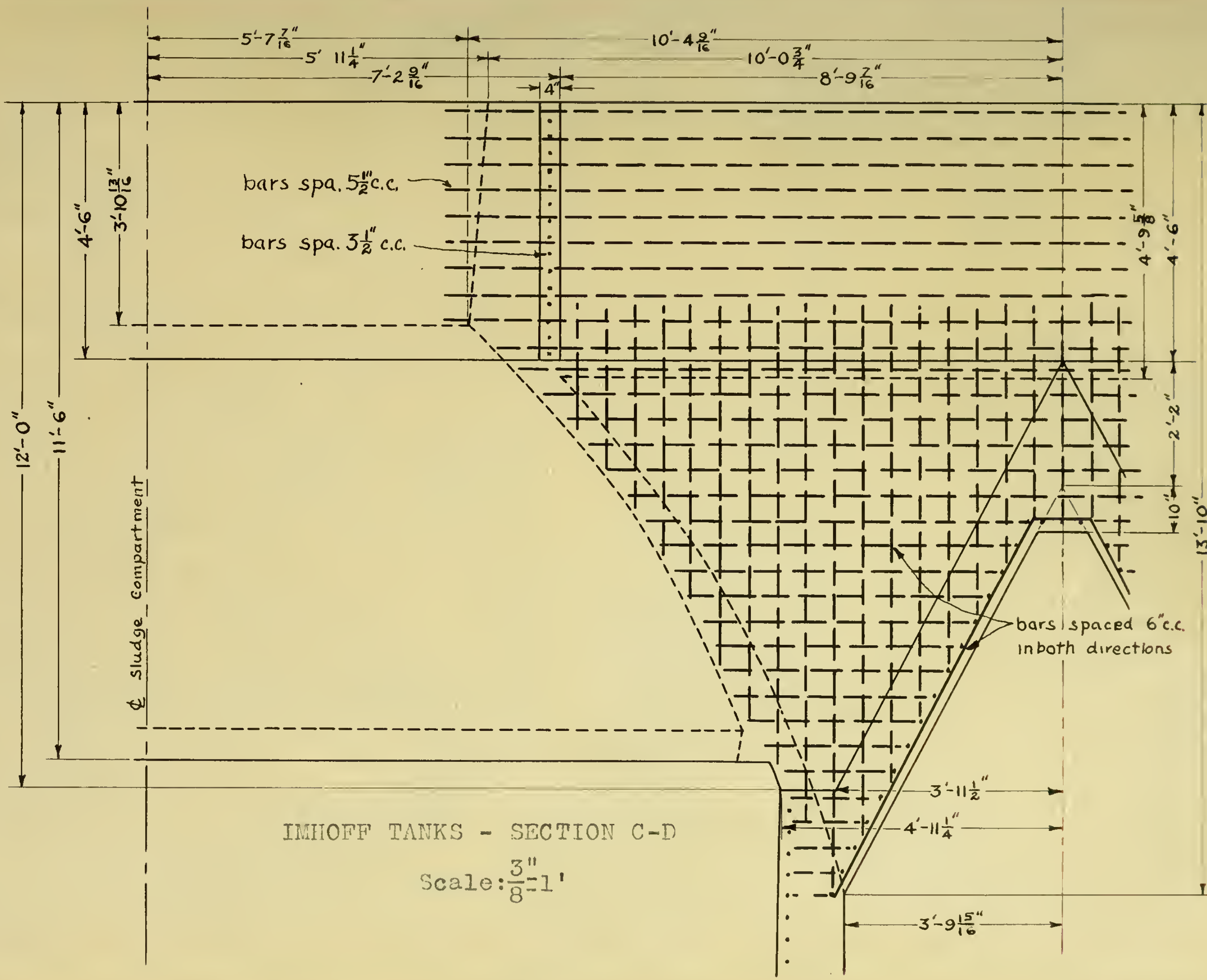
Scale: 1"=20'



IMHOFF TANKS

SECTION A-B

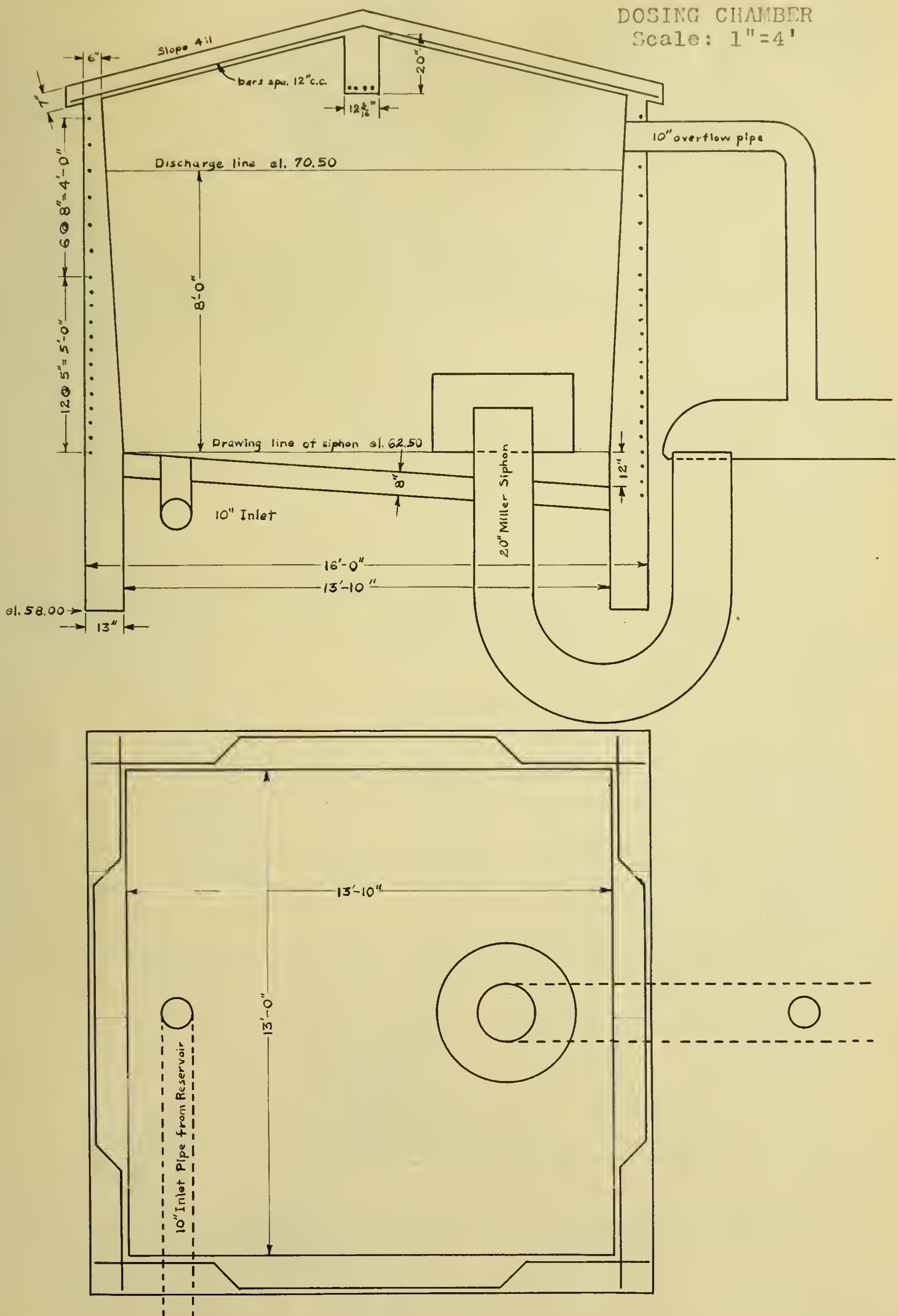
Scale: $\frac{3}{16}$" = 1'

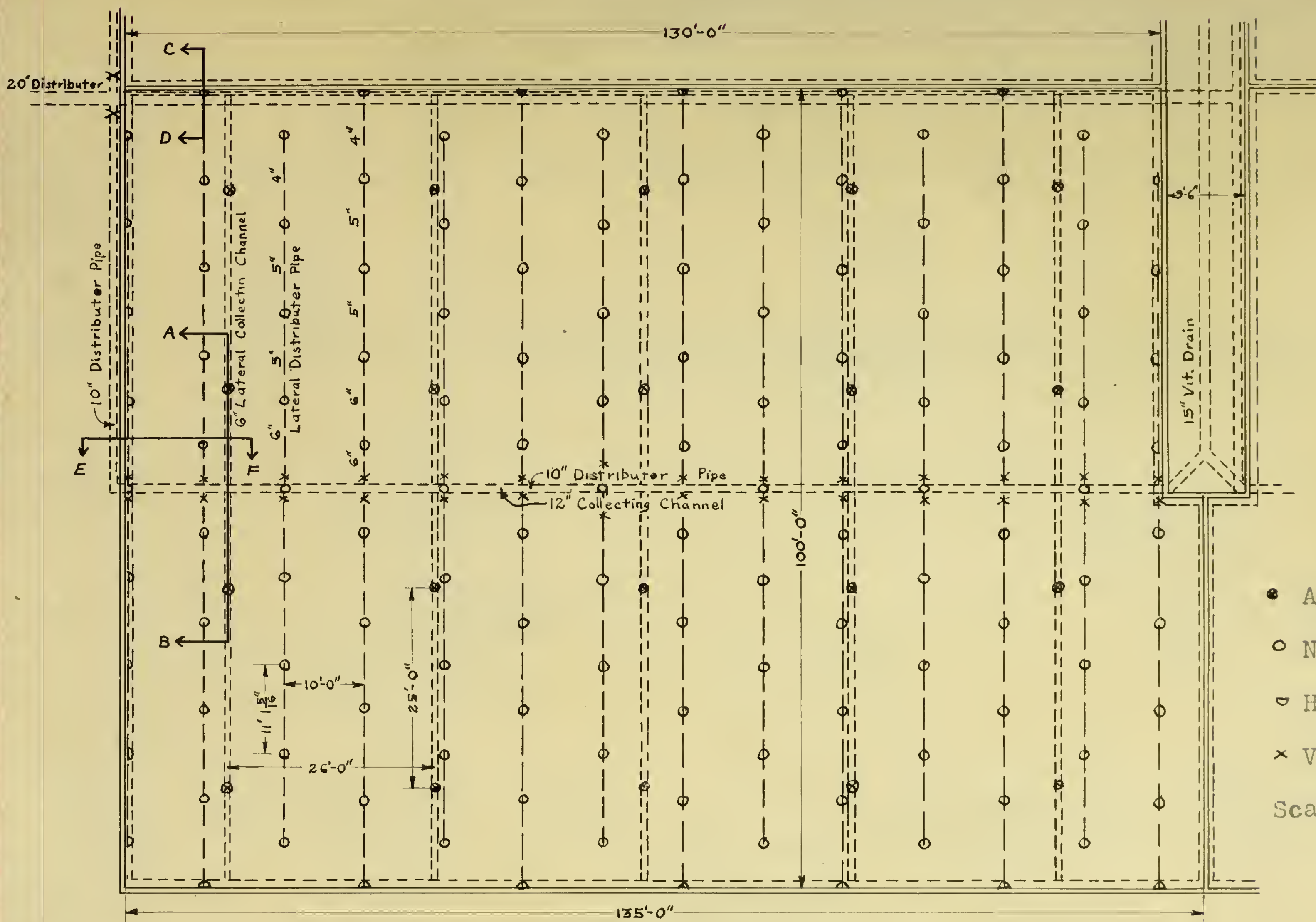


IMHOFF TANKS - SECTION C-D

Scale: $\frac{3}{8}"=1'$

DOSING CHAMBER
Scale: 1"=4'





FILTER BEDS

PLAN

S.W. $\frac{1}{4}$ of

Filter Area

Ledgend

● Air Vents

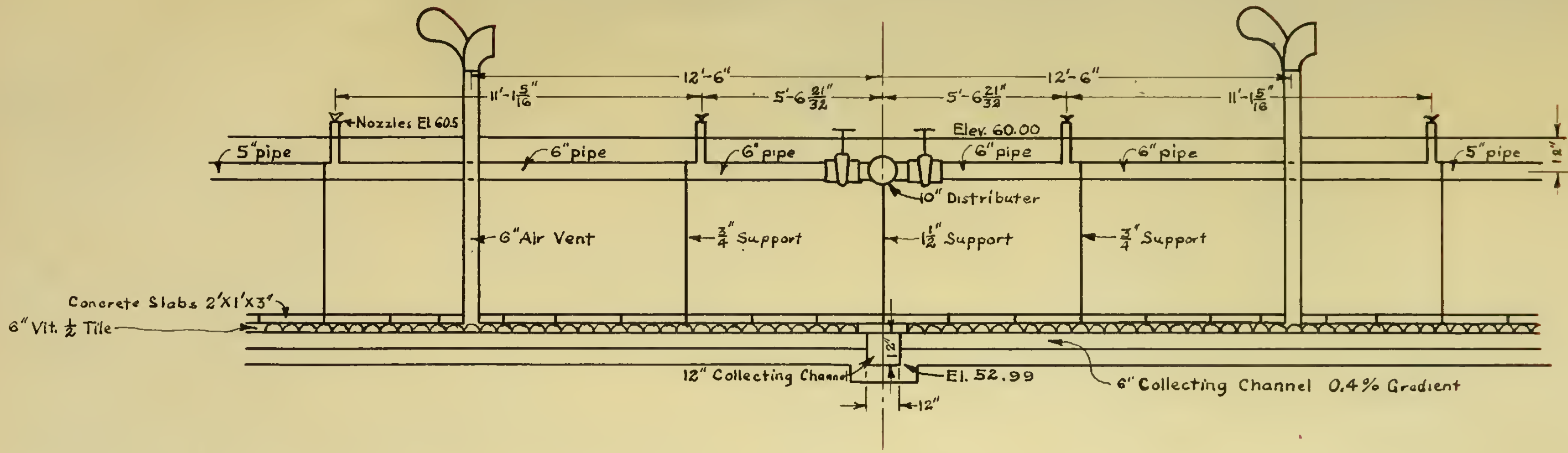
○ Nozzles

◡ Half Nozzles

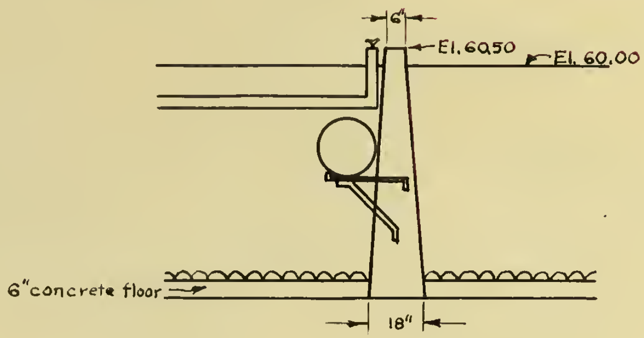
× Valves

Scale:1"=20'

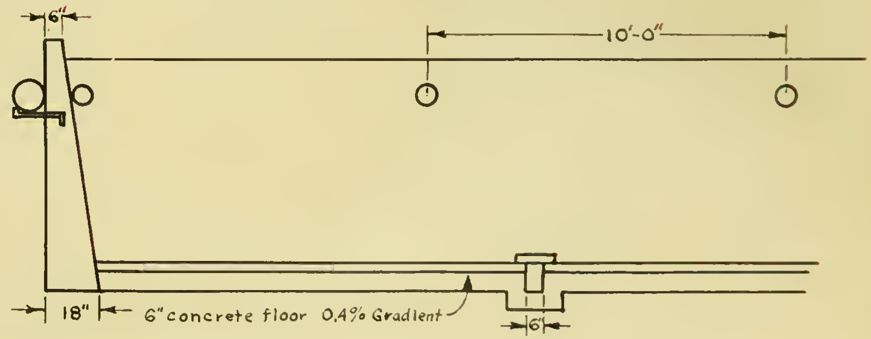
Section A-B



Section C-D



Section E-F



FILTER BEDS - SECTIONS

Scale: $\frac{3''}{16} = 1'$